

Panhandle Area, and Shoreline Area to identify remaining radiological hazardous substances at depth. The demarcation layer will consist of a permeable geosynthetic material and magnetic marking tape placed at the bottom of the soil cover. A demarcation layer will also be installed under the 4-foot-thick cover in the new wetlands. The soil cover, protective liner, and demarcation layer, in combination with ICs that are to be implemented throughout Parcel E-2 (as described in Section 2.6), will prevent unacceptable exposure of humans or wildlife to remaining contamination that exceeds the remediation goals (see Tables 2 and 3).

Table 1 identifies pertinent landfill closure requirements at Titles 22 and 27 Cal. Code Regs. The substantive provisions of these ARARs will serve as design criteria for the soil cover and protective liner for the Parcel E-2 Landfill and adjacent non-wetland areas. These ARARs pertain to various design elements, including geotechnical considerations, components of the soil cover and protective liner, surface water drainage, and erosion control. These design elements, and the pertinent ARARs that informed their design, are discussed in the Sections 3.5.1 through 3.5.4. The design basis of the soil cover for the new wetlands is discussed in Section 3.5.5 (the restoration design for the wetlands is further detailed in Section 3.9).

Figures 13 and 14 show the extent of the different cover types and conceptual cross sections. Section 3.5.2 describes the components of the proposed cover types.

3.5.1. Geotechnical Considerations

Golder Associates, under a subcontract to ERRG, performed geotechnical analyses in support of this DBR. The geotechnical analyses, which are presented in Appendix B, consist of the following:

- * A deterministic seismic hazard analysis to estimate the peak ground acceleration corresponding to the maximum credible earthquake (MCE) along the San Andreas Fault and to develop specifically matched acceleration time histories for seismic stability analysis.
- * An analysis of liquefaction potential using data collected during previous geotechnical investigations (from soil borings and cone penetrometer test [CPT] locations surrounding the Parcel E-2 Landfill).
- * An analysis of cyclic softening potential of the Bay Mud using data collected during previous geotechnical investigations (from soil borings and CPT locations at the southern perimeter of the Parcel E-2 Landfill adjacent to San Francisco Bay).
- * An analysis of the stability (under both static and seismic conditions) of three critical slopes in Parcel E-2 (each of which are adjacent to San Francisco Bay).
- * An analysis of the global stability of the Parcel E-2 Landfill (under both static and seismic conditions).
- * An analysis of the settlement potential of the landfill waste and underlying Bay Mud.

The geotechnical analyses were refined (relative to the Draft DBR) to incorporate additional data that were collected as part of the recent LFG survey (ITSI Gilbane Company, 2014). The following sections briefly summarize the results of the geotechnical analyses.

3.5.1.1. Analysis of Seismic Hazards

The final cover for the Parcel E-2 Landfill is designed to withstand the MCE, which is the design requirement under Title 22 Cal. Code Regs. § 66264.310(a)(5). The MCE is defined as "the maximum earthquake that appears capable of occurring under the presently known geologic framework." For Parcel E-2, the MCE matches the maximum historical earthquake (the 1906 San Francisco Earthquake), which is a moment magnitude (M_w) 7.9 earthquake located 12 kilometers from Parcel E-2 (to be conservative, a M_w of 8.0 was used in the analysis). The promulgated design requirements for seismic slope stability do not specify the method of estimating the ground motion for the MCE event. Accordingly, ground motion for the MCE event was estimated based on the current state of practice for seismic design. Specifically, the seismic hazard analysis consisted of the following steps to estimate the ground motion for the MCE event:

- * Ground motion prediction equations, which are empirical relationships developed from extensive ground motion databases for North America and other seismically active regions, were used to estimate the acceleration spectra for the MCE (damped by 5 percent).
- * Ground motion prediction equations were also used to estimate the median peak ground acceleration at Parcel E-2 following the MCE (0.29 times the acceleration due to gravity).
- * Four acceleration time history records for other areas that met the magnitude-distance criteria for the San Andreas Fault were reviewed and matched to the acceleration spectra for the MCE at Parcel E-2.

The results of the seismic hazard analysis are presented in Appendix E and were used in the liquefaction potential and seismic stability analyses. These analyses informed the design of the final cover for the Parcel E-2 Landfill.

3.5.1.2. Analysis of Liquefaction Potential

The Navy performed an initial analysis of liquefaction potential in 2004 (TIEML, 2004c). This analysis, which was based on data from soil borings and CPT locations surrounding the Parcel E-2 Landfill, concluded that sandy soil below and along the perimeter of the landfill is potentially liquefiable. The report also concluded that, because of the varying thickness and types of liquefiable soil at each location, the liquefaction is likely to be non-uniform (or discontinuous) across the site, which would be less damaging than liquefaction over a large continuous area. The Navy collected additional data, in support of this DBR, from soil borings and CPT locations along the southern perimeter of the Parcel E-2 Landfill (ERRG, 2013). Also, in conjunction with the recent LFG survey, an additional soil boring was installed in the western portion of the Parcel E-2 Landfill (ITSI Gilbane Company, 2014). Data from these

previous studies were used to refine the previous liquefaction potential analysis for this DBR. Based on the analytical results, it was concluded that:

- Sandy fill within and adjacent to the Parcel E-2 Landfill may liquefy during the MCE and affect the stability of the southern perimeter of the landfill (adjacent to San Francisco Bay).
- Lateral spreading is not considered a serious concern at this site because the potentially liquefiable zones appear to be discontinuous and isolated.
- Post-liquefaction settlement is unlikely to be critical to the overall stability of the landfill because it is estimated to be less than 6 inches.

The potential instability along the southern perimeter of the Parcel E-2 Landfill warranted an iterative slope stability analysis that is further detailed in Appendix E and briefly summarized in Section 3.5.1.4.

3.5.1.3. Analysis of Cyclic Softening of Bay Mud

Bay Mud samples were collected from soil borings along the southern perimeter of the Parcel E-2 Landfill. The samples were tested using several different methods to evaluate the strength of the Bay Mud during static and seismic (i.e., cyclic) loading conditions. Tests included constant rate of strain consolidation tests, static triaxial shear strength using consolidated undrained test conditions, a series of three cyclic triaxial shear strength tests, and post-cyclic triaxial shear strength tests (ERRG, 2013). The test results were used to analyze the potential for cyclic softening of the Bay Mud during an earthquake. The analysis showed that the Bay Mud has the potential to undergo cyclic softening during ground motion associated with the MCE. However, the post-cyclic triaxial shear strength tests on samples subjected to cyclic loading show that post-cyclic shear strength is similar to the original static shear strength. Therefore, the reduction of the strength of Bay Mud due to cyclic softening is expected to be small and potentially negligible. However, the presence of relatively soft Bay Mud, combined with the additional loading from the shoreline revetment, warranted a detailed evaluation of the stability of the southern perimeter of the landfill.

3.5.1.4. Stability Analysis of Southern Perimeter of Parcel E-2 Landfill

As described in Section 3.5.1.2, the potential exists for liquefaction of sandy fill materials underlying the perimeter of the Parcel E-2 Landfill during the MCE. Such liquefaction could affect the stability of the southern perimeter of the Parcel E-2 Landfill, including the shoreline revetment and final cover. To satisfy the requirements of Title 27 Cal. Code Regs. § 21145(a), a quantitative slope stability analysis was performed for three cross sections that are adjacent to San Francisco Bay and represent the most critical slopes of the final cover. Results of the iterative slope stability analysis, under both static and seismic conditions, determined that one layer of Tencate Miragrid® 22XT geogrid (or equivalent reinforcement with a combined long-term design strength of 11,266 pounds per foot) will be required to achieve the minimum factors of safety for geotechnical practice (1.3 for short-term conditions and 1.5 for long-term conditions).

The geogrid layer will be placed on the shoreline slope directly below the revetment material and will extend horizontally to an appropriate anchor point under the protective liner. The required total length of each geogrid layer, from the base of the slope to the upland anchor point, varies from approximately 65 to 110 feet (see design drawings C9 and C11 for further information). The geogrid layer will be installed in segments (perpendicular to the shoreline) in the following sequence:

1. Excavate soil and debris along the shoreline slope and upland area requiring reinforcement (excavated soil and debris will be managed as described in Sections 3.3.5, 3.3.6, and 3.3.7)
2. Install a continuous layer of geogrid from the base of the slope and to the upland anchor point (i.e., no splicing of the geogrid will be allowed between the base of the slope and upland anchor point)
3. Place and compact soil within the upland area to properly anchor the geogrid layer
4. Install revetment material

This general construction sequence is important because the short-term slope stability analyses (see Appendix E) identified a potential slope failure if the geogrid layers are not installed and anchored before placement of the revetment material. Also, as described in Section 3.3.1, additional excavation will be performed offshore of the revetment structure to ensure its integrity (during future remediation work in Parcel F) (see calculation brief A-2 for further information). The additional excavation will extend 6 feet horizontally offshore of the revetment structure and to depths ranging from 1.5 to 2.5 feet bgs (as shown on design drawings C9, C10, and C11 in Appendix B).

3.6.2. Soil Cover and Protective Liner for Parcel E-2 Landfill and Adjacent Non-Wetland Areas

The soil cover and protective liner for the Parcel E-2 Landfill and adjacent non-wetland areas were designed to meet the substantive requirements of pertinent ARARs (see Table 1). The soil cover and protective liner for the Parcel E-2 Landfill and adjacent non-wetland areas have the following components:

- * A foundation soil layer that is at least 2 feet thick [in accordance with Title 27 Cal. Code Regs. § 21090(a)(1)]
- * A protective liner that limits infiltration into the underlying material
- * A demarcation layer to identify remaining radiological hazardous substances at depth
- * A vegetative soil layer (at least 2 feet thick) that is capable of sustaining plant growth that will resist erosion [in accordance with Title 27 Cal. Code Regs. § 21090(a)(3)]

Soil cover material at depths greater than 0.5 foot below the final cover surface will be compacted to 90 percent or greater of the maximum dry density at or near optimum moisture, in accordance with ASTM International (ASTM)-modified proctor density testing. The upper 0.5-foot portion of the soil cover will

measures, such as the installation of raptor perches, are preferable as opposed to higher impact control measures, such as the use of poisons, to control burrowing animals at Parcel E-2.

3.5.5. Soil Cover and Demarcation Layer for Wetland Areas

The freshwater and tidal wetlands are being constructed to offset the loss of wetlands at Parcel E-2 and other areas at HPNS, in accordance with the substantive requirements of the federal Clean Water Act and the state of California's McAteer-Petris Act (see Table I). A liner will not be used in the new wetlands, so they function more naturally. To prevent exposure to contaminated material, the soil cover in the new wetlands will be 4 feet thick and will consist of 3 feet of hydric soil underlain by a demarcation layer (see Section 3.5.2.3) and a 1-foot-thick soil bridge (consisting of clean imported soil). Section 3.9 provides further information on the wetlands design basis.

3.6. SHORELINE REVETMENT

This section summarizes the basis of the draft RD for the shoreline revetment. The design criteria for the shoreline revetment along the boundary of Parcels E-2 and F include the RAOs and ARARs that pertain to soil and sediment, surface water, and groundwater presented in the ROD (Navy, 2012b). The revetment design will prevent unacceptable exposure to humans and wildlife from contaminants in shoreline sediment. The design requires the development of a San Francisco Bay Water Quality Monitoring and Protection Plan that specifies the monitoring requirements for nearshore excavations and revetment construction and BMPs that will control the discharge of contaminants into the bay. The plan will incorporate appropriate steps to comply with the specified discharge restrictions and minimize adverse impacts to waters of the bay. The design basis for the revetment adheres to standard practice for coastal engineering design, including pertinent USACE guidance (USACE, 2006), and address potential constructability issues and environmental controls for shoreline excavation.

A revetment is a facing of armor material, such as stone or concrete, which is intended to protect a shoreline from erosion. The primary physical design components of the revetment are the armoring material, the toe, the crest, and the filter layer. The armoring material is selected and sized based on the forces that will act on the structure, such as water currents, wave action, and gravity. The extent of the revetment, or the elevations of the toe and crest, is based on the expected high and low water conditions, significant wave heights, and wave runup on the structure. The filter layer is set between the armoring material and the underlying soil or engineered fill and is intended to allow water to pass while supporting the structure and preventing erosion of the underlying soil and sediment.

The design of the revetment for Parcel E-2 differs slightly from traditional revetment designs. These differences are related to its additional function to contain contaminated soil at Parcel E-2 and protect humans and wildlife. The following list summarizes the primary design basis elements that were used in developing the revetment design:

- The impact of anticipated maximum wave energy.
- The geotechnical stability of the revetment structure and shoreline slope.
- Water levels from tidal fluctuations and potential sea level rise.
- Prevention of human contact with potentially contaminated soil beneath the revetment.
- The need to minimize filling the tidal flats (present below 0 feet msl) with riprap.
- The future use of the area as open public space, and the possibility for foot traffic and vandalism along the revetment.
- Minimize negative impacts to the bay during construction.

The revetment will improve the shoreline of Parcel E-2 by replacing the existing shoreline with an armored, engineered revetment, meeting USACE design criteria (USACE, 2006). The revetment will prevent migration of contaminated sediment along the current shoreline and will integrate with the soil cover at the onshore portions of Parcel E-2. Sections 3.6.1 through 3.6.9 discuss the specific design basis elements and construction considerations.

The shoreline was inspected as part of the revetment design. The existing shoreline is partially armored with an unsupported revetment that maintains the shoreline in a relatively stable condition.

The revetment would be installed along approximately 1,800 feet of shoreline where Parcel E-2 meets Parcel F. The revetment will be approximately 35 feet wide³, and the crest elevation will be approximately +9 feet msl. In addition, a 3-foot-high concrete seawall will be incorporated into the crest of the revetment to accommodate wave runup. The sea wall was designed as an alternative to placing additional soil and armor rock to reach the design elevation (+12 feet msl), thereby providing an acceptable level of protection while minimizing the fill volume and associated weight of the shoreline revetment. This is an important consideration because, as discussed in Section 3.5.1, the relatively soft Bay Mud along the shoreline, combined with the additional loading from the shoreline revetment, affects the slope stability along the southern perimeter of the Parcel E-2 Landfill. The seawall would also act as a protective barrier along the shoreline walking path and act to hold back material that might erode from the upland soil cover.

A conservative approach for design of the revetment was taken to maximize its ability to prevent contaminated soil from migrating to the bay while remaining protective of humans and wildlife, considering the future use of the area. The long-term ongoing remedy developed in this DBR consists primarily of a soil cover and shoreline revetment over the site, which has been assessed for susceptibility to damage from seismic activity (see Section 3.5.1). The following sections and procedures for the

³ The width of the shoreline revetment adjacent to the proposed tidal wetlands appears narrower than 35 feet; however, as shown in design drawing C16 (specifically Section E) in Appendix B, clean soil within the tidal wetlands will be placed on top of the lower half of the revetment.

revetment design are based on the USACE “Design of Coastal Revetments, Seawalls, and Bulkheads” and “Coastal Engineering Manual, Parts I through VI” (USACE, 1995 and 2006). Figure 13 shows the proposed location of the shoreline revetment; Figure 15 provides a conceptual cross section of the revetment; and design drawing C10 in Appendix B provide the detailed cross sections along the revetment. The following sections summarize development of the revetment design.

3.6.1. Water-Level Ranges

The tidal ranges for Parcel E-2 were estimated from data published by the NOAA and the National Geodetic Survey (NOAA, 2009; National Geodetic Survey, 2012). Tidal data serve as the basis for design of a revetment as a primary component in calculating the crest elevation and the extent of the structure. All data were corrected to NGVD 1929, the universal vertical datum selected for this design package.

The tidal range between the mean higher high water (MHHW) and the mean lower low water (MLLW) was approximately 6.73 feet for the tidal epochs of 1960 through 1978 (NOAA Hunters Point Tidal Benchmark and Datum). The MHHW and MLLW are defined as the mean of the higher high water height and the lower low water height of each tidal day observed over the tidal datum epoch. When adjusted for msl based on NGVD 1929, the MHHW is +3.61 feet above msl and the MLLW is -3.12 feet below msl. Tidal ranges are generally referenced to MLLW; however, elevations are referenced to msl in this design to remain consistent with overall site elevations and the surveys completed.

Also of significance for the revetment design is the determination of the highest water levels expected. Tidal data were obtained to assess the extreme high and low water events for the tidal epochs described above. An extreme high tide of 6.7 feet above msl can be expected for the project location and is associated with a 100-year return period (USACE, 1984). Appendix A, calculation brief A-11, includes the tidal range calculations and adjustment factors. Table 10 summarizes the primary tidal data elevations used for the revetment design and calculations.

The potential for an increase in the sea level elevation as a result of atmospheric warming has been considered in the design of the revetment. The paragraphs below briefly summarize information on potential increases in the global sea level over the next 100 years.

The Intergovernmental Panel on Climate Change (IPCC) estimated potential increases in the global sea level over the next 100 years. The following excerpt from Church et al. (2008) summarizes the most recent IPCC estimates of global sea level rise (which range from 0.3 feet to 2.9 feet): “The IPCC provides the most authoritative information on projected sea-level change. The IPCC Third Assessment Report of 2001 (Church et al., 2001) projected a global-averaged sea-level rise of between 20 and 70 centimeters (cm) between 1990 and 2100 using the full range of IPCC greenhouse gas scenarios and a range of climate models. When an additional uncertainty for land-ice changes was included, the full

range of projected sea-level rise was 9–88 cm. For the IPCC's Fourth Assessment Report (Meehl et al., 2007), the range of sea-level projections, using a larger range of models, is 18–59 cm (90% confidence limits) over the period from 1980–1999 to 2090–2099 (Meehl et al., 2007).²⁹

The estimates from the IPCC's Fourth Assessment Report used data published in 2005 or earlier and, because of uncertainty in the available information, did not account for potential increases in sea level due to changes in ice sheet dynamics (i.e., rapid loss of ice sheets that could contribute to sea level rise). More recent research has yielded a wide range of estimated sea level rise over the next 100 years (between 1.8 to 6.5 feet) that accounts for changes in ice sheet dynamics but reflects the significant uncertainty in such projections (National Research Council, 2010).

Based on the information summarized above, a contingency of up to a 3-foot increase in sea level was considered in designing the crest elevation, which is discussed in Section 3.6.6.

3.6.2. Wind and Wave Dynamics

Wave height is an element of the revetment design basis that depends largely on the velocity of wind over the water, the duration of the sustained wind, and the available wind fetch (i.e., the uninterrupted over-water distance where wind can affect the water surface). The greatest 2-minute sustained peak wind speeds that might affect generation of waves for the site are anticipated to be from a direction of 160 degrees (clockwise) from north at 49 mph and have been estimated to have a 100-year return period. Unsustained wind gusts of short duration do not significantly affect the formation of waves. Appendix A, calculation brief A-12, summarizes the determination of the wind dynamics for the design.

The proposed revetment area is directly exposed to waves generated by winds blowing from between 143 degrees (clockwise) from north and 176 degrees from north as shown in the fetch distance and wind parameters calculation brief (A-12). Fetch distances in the other cardinal directions are restricted by significant landmasses. The design wind speed should be appropriate to the design fetch distance. The fetch for winds blowing from 160 degrees (clockwise) from the north extends from Coyote Point, which is over 10 kilometers (6.2 miles) from the Parcel E-2 shoreline. However, not all of these distances are available at all points along the shoreline. Appendix A, calculation brief A-12, summarizes the available fetch distances for Parcel E-2.

A series of deep-water wave heights can be calculated for given wind and fetch parameters available for a location. Appendix A, calculation brief A-13, summarizes the anticipated wave heights by appropriate cardinal direction for the site. The highest calculated wave height is considered the significant wave and is used in the design of a revetment or other coastal structures. The highest calculated significant wave height anticipated for the site would be from 160 degrees (from the north), at a wind speed of 41 mph, with a height of 4.1 feet associated with the 100-year return period winds. This maximum anticipated significant wave of 4.1 feet serves as the design wave in calculations throughout the design.

The design wave, as summarized above, is an open water wave that would break before it reached the shoreline and the revetment. It is used in this design and provides a conservative estimate of the wave that can be expected to affect the revetment. Generally, a wave will break when it reaches a water depth equal to or less than approximately 128 percent of its height (USACE, 2006). Based on this ratio of wave height to depth, the design wave will break in 5.3 feet of water (see Appendix A, calculation brief A-14). The open water wave has been used in this design because it is conservative; however, most waves will break before they reach the revetment, so the actual wave energy along the revetment will be less.

3.6.3. Selection of Armor Material

The revetment can be constructed from a wide variety of materials, including stone, concrete, or prefabricated mats and blocks. Potential materials were screened and selected based on strength, availability, cost, and constructability. Special consideration was also given to the future use of the area in selecting the material. It was determined that the bulk of the revetment structure should be constructed using stone, as it will appear more natural, and because stone-based options are flexible and are able to withstand minor damage without compromising strength and function. Revetments constructed from stone can also be repaired more easily than can structures made from prefabricated materials. As described previously, a concrete seawall will be incorporated into the crest of the revetment to accommodate wave runup.

Three options are available for the primary natural stone: (1) multiple layers of angular uniform-sized rock (quarrystone), (2) graded rock of sizes between upper and lower limits (riprap), and (3) layers of subrounded to rounded boulders (field stone). The primary disadvantage to revetments constructed of fieldstone is that they have considerably less strength than revetments constructed of more angular quarrystone or riprap material of the same weight. Obtaining larger fieldstone needed for a revetment could be difficult, and placement of this material is costly. For this reason, fieldstone was not considered.

When comparing the strengths of randomly placed quarrystone versus randomly placed riprap, a greater thickness of quarrystone is needed to achieve the strength of a thinner layer of riprap. The quarrystone option would likely have a greater cost than riprap for a comparable strength. For this reason, uniform quarrystone was not considered suitable. Additionally, it has significant void space between rocks, which could pose a trip hazard to potential foot traffic given the anticipated future use of the site as a park.

Riprap is not recommended for revetments with sustained exposure to waves larger than 5 feet. The significant wave for this design was estimated at 4.1 feet, and a natural riprap material was deemed the most appropriate option. This design incorporates a randomly placed natural riprap, which is typically composed of two layers of the selected median size armor stone.

3.6.4. Revetment Slope Selection

For Parcel E-2, the two primary considerations in selecting the revetment slope are (1) the geotechnical stability of the revetment structure and shoreline slope, and (2) the need to integrate the revetment structure with the surface elevations of the proposed final cover (which, as described in Sections 3.3 and 3.4, will include onsite consolidation of more than 122,000 cubic yards of material before placement of the final 2-foot-thick soil cover). A uniform revetment slope of 3H:1V will be stable under the seismic design conditions (summarized in Section 3.5.1 and detailed in Appendix E), and integrates with the proposed elevations of the final cover. The proposed 3H:1V slope is also similar to the conditions along the Parcel E-2 shoreline prior to the Phase 2 TCRA at the PCB Hot Spot Area; which, as described in Section 2.6, removed significant volumes of contaminated soil and debris and resulted in more gradual shoreline slopes.

A secondary consideration in selection of the slope is the assessment of the existing shoreline slope and the amount of soil and sediment that would need to be excavated or filled to achieve the prescribed slope. The existing slope along the extent of the proposed revetment (which was substantially reduced following removal of contaminated soil and debris during the Phase 2 TCRA at the PCB Hot Spot Area) varies between about 20H:1V and 10H:1V, depending on location, with the average slope being about 14H:1V. A uniform revetment slope of 3H:1V increases the existing slope along the shoreline, which reduces the total amount of excavation necessary along the revetment.

In addition, a uniform slope along the revetment is desirable for ease in construction and to maintain uniform rock gradation throughout the structure. More steeply sloped riprap revetments are inherently less stable, and larger rock sizes are needed to achieve the same strength as less steeply sloped structures. If varying slopes are used along the revetment, varying riprap sizes are needed to maintain the stability, which can increase the risk of failure and can complicate operations and maintenance of the structure. Therefore, a uniform slope has been used in this design basis.

Figures 13 and 14 show a plan view and details of the revetment extents and final grade. Figure 15 shows a typical cross section of the revetment.

The shoreline will be excavated for construction of the revetment, and most of the revetment will be placed on Bay Mud or clean import fill. Approximately 10,700 bank cubic yards of soil and sediment will be excavated along the shoreline to achieve the prescribed slopes for the revetment (Appendix A, calculation brief A-3). Excavated sediment may be consolidated on site following radiological screening, as described in Sections 3.3.5 and 3.3.7.